Shear Strength of Post-Tensioned Grouted Keyed Connections

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The shear transfer strength of post-tensioned grouted keyed connections was studied by conducting direct shear tests on shear couplets (push-off tests). The variables studied include different levels of prestressing and distribution of shear keys along the height of the connection. The shear strength of each connection was evaluated using both the ACI and PCI shear-friction approaches and compared with experimental results. Based on the experimental investigations, an empirical equation that depends on the grout strength and prestress force across the connection is proposed for evaluating the shear strength of post-tensioned grouted keyed connections.

Continuous connections provide three-dimensional stability to precast concrete framed structures under lateral loads. There are several methods of creating continuity in precast concrete construction. One such method, which is gaining wide acceptance due to its efficiency, is post-tensioning.

A post-tensioned grouted keyed connection suitable for building frames, as shown in Fig. 1, has the neat appearance of a monolithic concrete connection unlike a simple connection with a projecting corbel. Grouted keyways absorb dimensional tolerances characteristic of precast concrete elements.

The pertinent features of this connection have been presented elsewhere.[1] The details of the connection illustrated in Fig. 1 are merely a suggestion. This paper presents the details of push-off tests and the shear strength of this type of connection. A numerical example in Appendix B shows a comparison of the prestressing force required at a connection using the ACI, PCI and proposed methods of calculation.

EXPERIMENTAL PROCEDURE

Push-off specimens were chosen for the study of shear transfer behavior and determination of shear strength, since these types of tests were successfully employed by earlier investigators for similar studies.[2-4] High strength bars were chosen for prestressing because of their efficien-

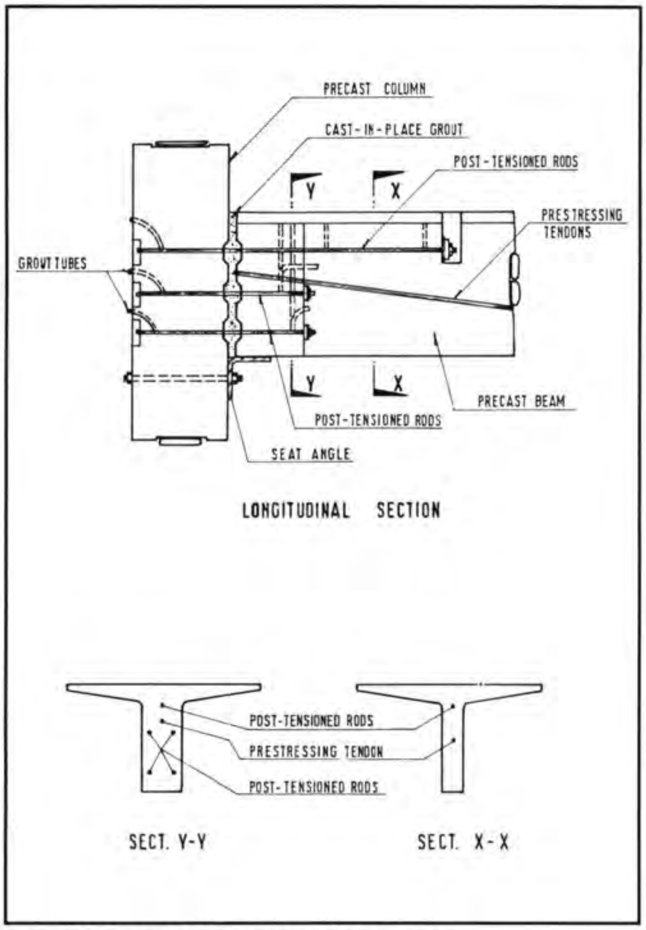

Fig. 1. Details of post-tensioned connection.

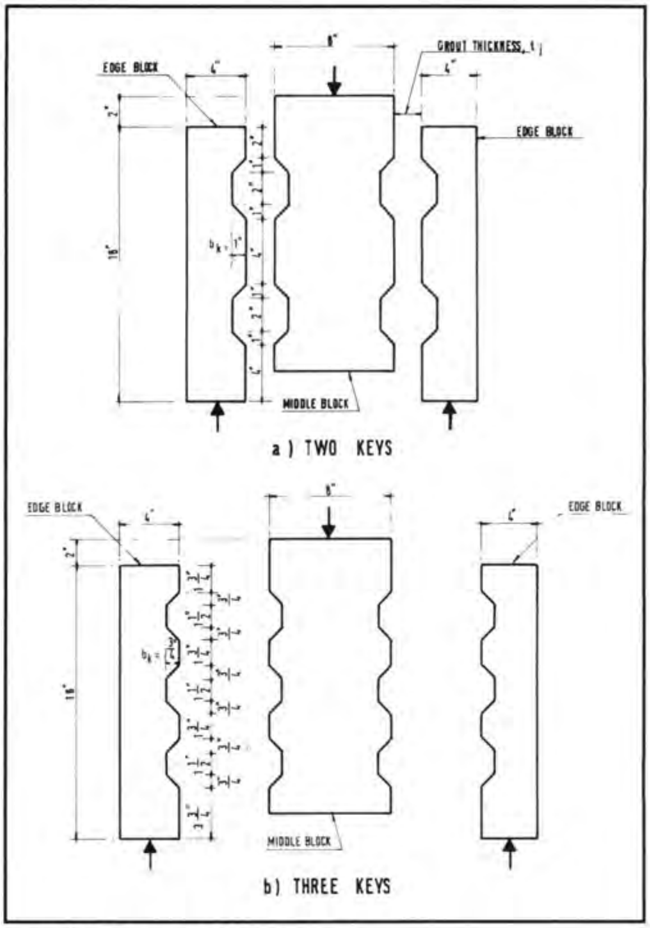

Fig. 3. Configuration of shear keys.

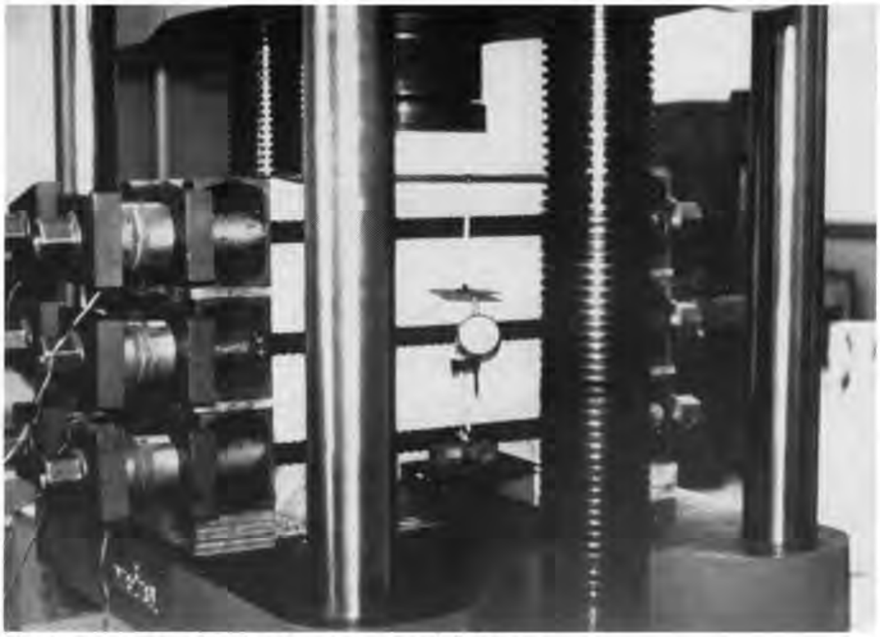

Fig. 2. Test setup showing specimens and dial gage.

cy for prestressing short lengths.

As shown in Figs. 2 and 3, each specimen assembly consists of one concrete middle loading block and two concrete edge reaction blocks with grout joints between them. Twenty-six specimens had a grout joint width (minimum dimension of joint) of 1 in. (25.4 mm). Both the edge and middle blocks were 8 in. (203 mm) thick.

No reinforcing bars were used in these concrete blocks and no reinforcement was placed across the joints. The mix used for the grout was found to have 200 percent flow.[5] Iced water between 45°F and 55°F (7°C and 13°C) was used for mixing grout in order to reduce the water required for the desired consistency and retard the setting time.

The middle block was kept 2 in. (51 mm) above the edge blocks by seating it on a wooden block of size 12 x 8 x 2 in. (305 x 203 x 51 mm). The grout mix was passed through a plastic funnel and the entire joint for each specimen assembly was done in three mixes. For each mix, a pair of 2 in. (51 mm) cube compression specimens were cast. The joint was rodded using a tamping rod in order to mix the layers of grout and expel possible air pockets. The specimen assembly, concrete cylinders and grout cubes were all later cured by moist curing for 28 days under an identical environment.

Post-Tensioning

Threadbars of 1 in. (25.4 mm) nominal diameter were used for prestressing. These bars have a continuous rolled-in pattern of thread-like deformations along the entire length. Threadbars have an ultimate strength of 150,000 psi (1035 MPa) and are made of hot rolled, proof stressed, alloy steel conforming to ASTM A 722.[6] Assembled with plate anchorages and nuts at the ends, each bar made a high strength long bolt.

As shown in Fig. 2, the bars were external to the specimen. External prestressing was used to exclude the dowel action of prestressing bars. Mast[7] had also pointed out the need for testing specimens with reinforcement external to the concrete. The specimen assembly was post-tensioned using a maximum of six bolts, and each bolt was equipped with a load cell between bearing plates at one end (see Fig. 2). One load cell was placed on each prestressing bolt and the prestress was applied immediately before testing. The prestressing was done in four equal stages by turning the nuts on the end of the bar opposite to the load cell.

Testing and Instrumentation

After initial prestressing, the specimens were lifted on to the bottom platen of a universal testing machine. Once centered in the machine, prestressing to the desired level was completed. The specimens were lightly gripped between the lower and upper platens of the machine while turning the nuts to prevent the specimen from tilting.

Dial gages were mounted on both the front and back of the specimens to measure the relative displacement between the bottom surface of the middle block and the top surface of the edge blocks (the slip). Fig. 2 illustrates the dial gage arrangement on one side of the specimen.

The load was gradually increased to 50 kips (222 kN) as the dial gages were monitored. The load was then increased in increments of 10 kips (45 kN) and dial readings were recorded. The loads at which first slip and first cracking occurred were recorded. The tension in the post-tensioning bars was monitored at intervals of 25 kips (111 kN). The bar tension remained constant up to loads approaching ultimate and, consequently, was never adjusted. Cracks were observed and marked at all load stages. At loads approaching ultimate, the slip increased rapidly and the dial gage readings were terminated. The specimen was loaded until it failed to accept further load. The ultimate shear is defined as the maximum shear carried by the specimen during the test.

TEST VARIABLES AND SPECIMEN DETAILS

Details of the specimens tested in this investigation are given in Table 1. The three test series, planned to include the different experimental variables, are designated as A, B and C. Specimens of Series A had two shear keys while the specimens of Series B had three shear keys. Series C had no keys. The two and three shear key configurations are shown in Figs. 3(a) and 3(b). The total key areas, B (sum of the products of key height and joint thickness), in two key and three key configurations were 64 and 72 sq in. (0.041 and 0.046 m^2), respectively. Key areas were kept as close to equal as possible since the objective was to study the effect of distribution of keys over the height of joint.

Specimens A-8, A-9, B-8, and B-9 were prestressed to have a triangular distribution of prestress with zero psi at the top, and 1000 psi (6.9 MPa) at the bottom (hereafter referred to as Type 1 prestressing). Specimens A-10, A-11, B-10 and B-11 were prestressed to have a triangular distribution of prestress with 1000 psi (6.9 MPa) at the top and zero psi at the bottom (hereafter referred to as Type 2 prestressing). All other specimens had a uniform distribution of prestress.

Table 2 shows the average compressive strength of three companion cylinders for each specimen. The weight of grout was 126 lb per cu ft (2.02 kN/m^3) in contrast to the weight

Table 1. Details of test specimens.

Serial number (1)	Specimen (2)	Number of keys (3)	Joint thickness, in. (4)	Prestress, psi (5)
1	A-1	2	2	0
2	A-2	2	2	400
3	A-3	2	2	400
4	A-4	2	2	600
5	A-5	2	2	600
6	A-6	2	2	800
7	A-7	2	2	800
8	A-8	2	2	0 top, 1000 bottom
9	A-9	2	2	0 top, 1000 bottom
10	A-10	2	2	1000 top, 0 bottom
11	A-11	2	2	1000 top, 0 bottom
12	A-12	2	1	800
13	A-13	2	1	800
14	B-1	3	2	0
15	B-2	3	2	400
16	B-3	3	2	400
17	B-4	3	2	600
18	B-5	3	2	600
19	B-6	3	2	800
20	B-7	3	2	800
21	B-8	3	2	0 top, 1000 bottom
22	B-9	3	2	0 top, 1000 bottom
23	B-10	3	2	1000 top, 0 bottom
24	B-11	3	2	1000 top, 0 bottom
25	B-12	3	2	1000
26	B-13	3	2	1000
27	C-1	0	2	800
28	C-2	0	2	800

Metric (SI) conversion factors: 1 in. = 25.4 mm; 1000 psi = 6.9 MPa.

of normal weight concrete of 150 lb per cu ft (2.40 kN/m³). The grout can be considered as lightweight concrete since concretes having weights not exceeding 115 lb per cu ft (1.84 kN/m³) are classified as structural lightweight concrete.[8] Typical stress-strain relations of concrete and grout are shown in Fig. 4.

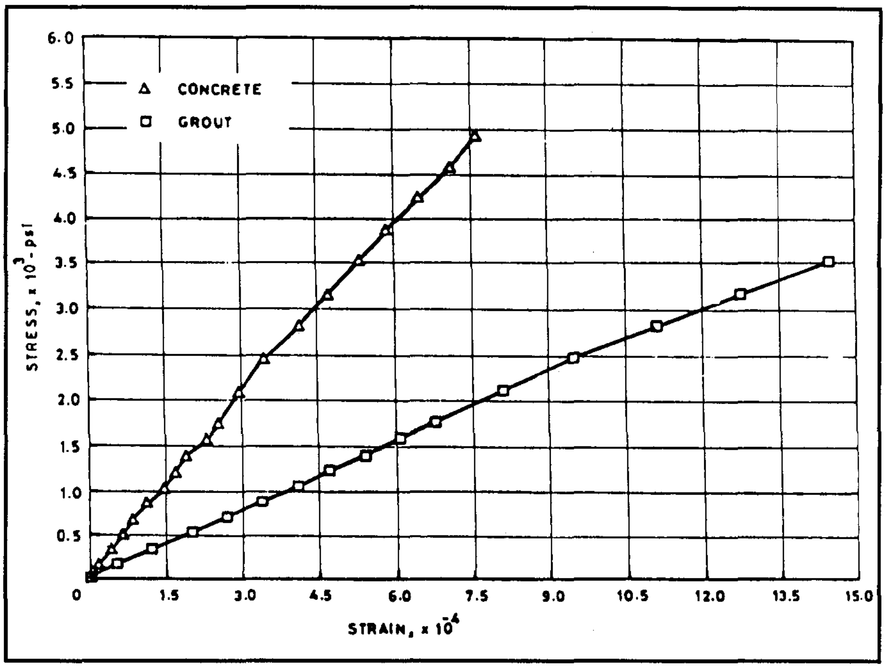

Fig. 4. Stress-strain relations for concrete and grout.

SHEAR STRENGTH

The values of ultimate shear force observed during the tests are listed in Table 2. The observed shear strength is expressed as a nominal ultimate shear stress over the area of the 16 x 8 in. (406 x 203 mm) shear plane. The variation of shear strength with uniform prestress for specimens with two and three keys are shown in Fig. 5. The variation of shear strength with prestress including data points for specimens with variable prestress (Type 1 and Type 2) are shown in Fig. 6. An average prestress of 500 psi (3.5 MPa) is considered in the case of specimens with variable prestress. The data points are fitted by linear regression.

The shear strength is seen to increase with increasing prestress in both cases of specimens with two and three keys. The rate of increase of shear strength with increase of prestress is higher for specimens with three keys than it is for specimens with two keys. The following are the equations of the lines of fit shown in Figs. 5 and 6.

(a) Specimens with uniform prestress:

(i) Shear strength of specimens with two keys

$$v_n = 404.60 + 0.56f \qquad (1)$$

(ii) Shear strength of specimens with three keys

$$v_n = 434.90 + 0.74f \qquad (2)$$

(b) Specimens with uniform and variable prestress:

(i) Shear strength of specimens with two keys

$$v_n = 490.40 + 0.64f \qquad (3)$$

(ii) Shear strength of specimens with three keys

$$v_n = 536.80 + 0.46f \qquad (4)$$

where

v_n = shear stress, psi

f = prestress, psi

The correlation coefficients of Eqs. (1) through Eq. (4) are 0.99, 0.96, 0.53 and 0.92, respectively.

The above equations, except Eq. (3), indicate a nearly perfect positive linear association with the experi-

Table 2. Comparison of shear strengths.

Specimen (1)	Strength of concrete, psi (2)	Strength of grout, psi (3)	Obs. ult. shear V, kips (4)	Shear strength, psi ACI (5)	PCI (6)	Experimental (7)
A-1	7797	6133	60.0	156.63	156.63	468.75
A-2	7975	5040	78.75	400.00	400.00	615.23
A-3	7166	4047	81.00	400.00	400.00	632.81
A-4	7950	4392	98.25	600.00	600.00	767.58
A-5	8770	4232	94.25	600.00	600.00	736.33
A-6	8611	3675	108.25	800.00	800.00	845.70
A-7	7142	5573	109.00	800.00	800.00	851.56
A-8	7738	5894	85.75	500.00	500.00	669.92
A-9	8145	4092	72.75	500.00	500.00	568.36
A-10	7944	6471	115.25	500.00	500.00	900.39
A-11	8068	7213	116.00	500.00	500.00	906.25
A-12	7573	7600	144.25	800.00	800.00	1126.95
A-13	7950	7777	161.50	800.00	800.00	1261.72
B-1	8351	6375	76.50	159.69	159.69	597.66
B-2	8363	6017	91.25	400.00	400.00	712.89
B-3	8304	6133	105.00	400.00	400.00	820.31
B-4	8439	5263	108.25	600.00	600.00	845.70
B-5	8003	5696	107.75	600.00	600.00	841.80
B-6	8097	6183	131.75	800.00	800.00	1029.30
B-7	8439	6175	124.25	800.00	800.00	970.70
B-8	7814	5838	118.25	500.00	500.00	923.83
B-9	7685	5654	100.00	500.00	500.00	781.25
B-10	7785	5175	115.75	500.00	500.00	904.30
B-11	8074	5692	116.25	500.00	500.00	908.20
B-12	7749	5879	157.25	800.00	1000.00	1228.52
B-13	8268	5700	152.50	800.00	1000.00	1191.41
C-1	7478	6492	58.75	480.00	600.00	458.98
C-2	7396	6318	74.75	480.00	600.00	583.98

Metric (SI) conversion factors: 1000 psi = 6.9 MPa; 1 kip = 0.4537 kN.

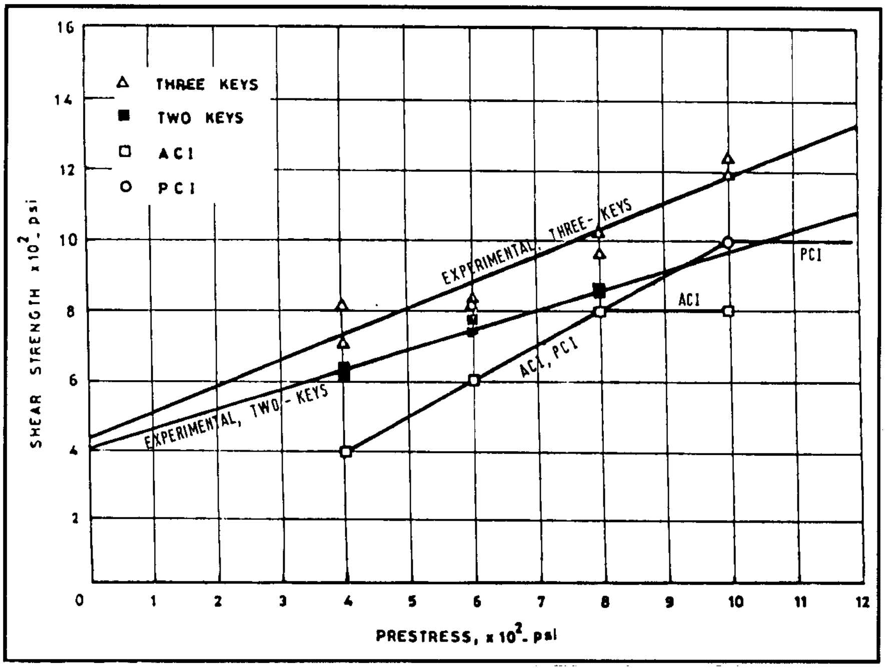

Fig. 5. Variation of shear strength with uniform prestress.

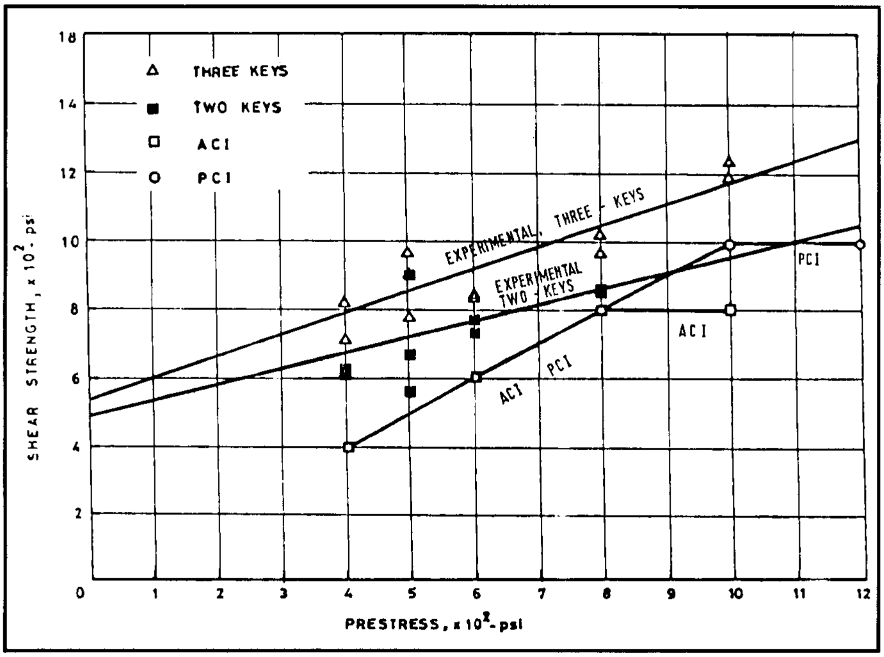

Fig. 6. Variation of shear strength with uniform and variable prestress.

mental data. However, the experimental data is limited and further data is needed to verify the relations between prestress and shear strength, particularly outside the test range. The differences in the prediction of shear strength between using Eqs. (1) or (3) and Eqs. (2) or (4) are only in the order of 3 percent. Hence, the shear strength of specimens with variable prestress can be determined using Eqs. (3) or (4).

The variations of shear strength with number of keys at different levels of uniform prestress across the connections are presented in Fig. 7. The shear strength of the three-key connection is higher for all prestress levels. The rate of increase is similar with 400 and 800 psi (2.8 and 5.3 MPa) prestress levels while there is a slight difference in the rate of increase for the 600 psi (4.1 MPa) level.

The ratio of key area, B, to total joint area, A_{cr}, (B/A_{cr}) is practically the same for the connection with two and three keys. Thus, a distribution of the total key area along the vertical height of the joint appears to be beneficial with regard to the shear strength of the connection. Due to the limited experimental data, the rate of increase of shear strength with increase in number of keys cannot be extrapolated with confidence outside the range of two and three keys.

Rizkalla et al.[9] have reported that the shear key configurations considered in their study had an insignificant effect on the behavior and capacity of connections. They have, however, found that an increase in ultimate shear resistance was significant in the case of large shear key connections with increasing prestress. This is probably due to the effect of a high B/A_{cr} value (0.49) in the case of large keys when compared to small shear key connections ($B/A_{cr} = 0.39$).

Earlier research on grouted shear key connections in large panel construction has indicated the preference to use keyed joints with B/A_{cr} values less than 0.5 to avoid failure of the panel instead of the joint.[10] This is quite relevant in the case of beam-column connections as well. In several of the three-keyed specimens of the current investigation, a single diagonal crack was found to develop in the middle block, but it did not grow in size as the loading was increased. The dimension of key along the thickness of the joint (width of key, b_k) is recommended to be not less than 0.5 in. (12.7 mm).[11] Following these recommendations and other practical considerations, a four-key configuration is the practical maximum for the shear transfer length of 16 in. (406 mm) adopted in this investigation.

The shear strength of the specimens with no shear keys can be predicted based on a coefficient of friction, μ, the prestress normal to the connection, f, and the joint area resisting shear, A_{cr}, as follows:

$$V_u = \mu f A_{cr} \tag{5}$$

Using the averge value for V_u for Specimens C-1 and C-2, a coefficient of friction of 0.65 was computed. This value compares well with the coefficient of 0.62 reported by Rizkalla et al. in previous studies.[9-12]

The beneficial effect of shear keys is evident by a comparison of V_u values for Specimens C-1 and C-2 to those values for Specimens A-6, A-7, B-6 and B-7 (see Table 2). The ratio of the average values of the C series specimens to the average values of the A and B series specimens is 0.57. The comparable value reported by Rizkalla et al.[9] is 0.62. It should be noted that the definition of V_u utilized in the present study is more comparable to the definition of "maximum load" in the work by Rizkalla et al. It is also noted that these favorable comparisons can be made regardless of significant differences in test specimen configurations and materials utilized in the two studies.

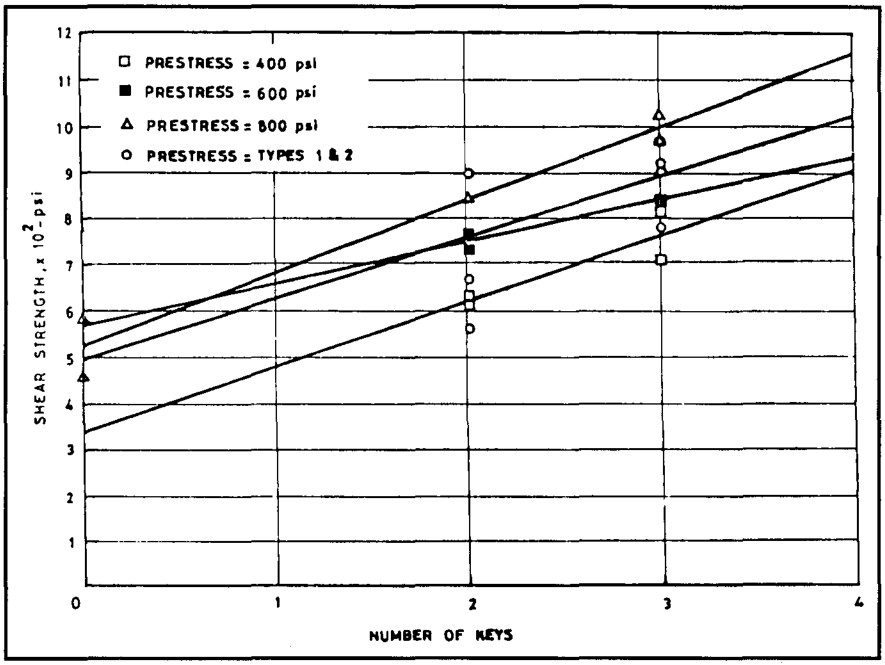

Fig. 7. Variation of shear strength with number of keys.

Comparison With ACI Design Equation

Precast connections are designed using the shear-friction concept that has been accepted mainly by the ACI and the PCI. The recommendations are based on the results of experimental investigations reported on different types of precast concrete connections.[2,4,7,13]

The ACI recommendation for the nominal shear strength is:

$$V_n = A_{vf} f_y \mu \qquad (6)$$

where

A_{vf} = area of reinforcement nominally perpendicular to assumed crack plane

f_y = yield strength of reinforcement

μ = coefficient of friction

μ = 1.4 λ for monolithically cast concrete

μ = 1.0 λ for concrete cast against hardened concrete with roughened surface

μ = 0.6 λ for concrete cast against hardened concrete not intentionally roughened

μ = 0.7 λ for concrete anchored to structural steel

in which

λ = 1 for normal weight concrete

λ = 0.85 for sand lightweight concrete

λ = 0.75 for all lightweight concrete

As permitted by the ACI, the prestress force is substituted for the product $A_{vf} f_y$ in Eq. (6) and the shear strength expressed as a nominal shear stress. The grout is considered as a normal weight concrete, and hence, a value of μ of 1.0 is used for all computations. The maximum permitted value of V_n (the greater of 0.2 $f'_c A_{cr}$ or 800 A_{cr} in lb) is also considered using the appropriate values of compressive strengths of grout, f'_c, and the cross-sectional area, A_{cr}, of the joint.

The resulting values are tabulated in Column 5 of Table 2. Fig. 6 shows the comparison of experimental values of shear strength with values obtained using the ACI shear-friction relation. For Specimens A-1 and B-1, without prestress, the strength has been calculated using corresponding grout strengths and the conventional shear formula ($v_c = 2\sqrt{f'_c}$).

The ACI Code equation is found to underestimate the shear strength of grouted shear key connections subjected to post-tensioning normal to the transfer plane in both two-key and three-key configurations. The differences between the ACI predictions and experimental results will be more pronounced if the grout is assumed to be lightweight concrete and there is no dowel action present in the experimental results. Both considerations would result in a lower value of μ in Eq. (6).

The ACI Code limits the maximum shear strength that can be carried across a given shear plane to 800 psi (5.52 MPa) to prevent congestion of reinforcement.[14] This problem does not arise in post-tensioned connections and, considering the high grout strength, a prestress level in excess of 1000 psi (6.9 MPa) is reasonable. The shear transfer strength at 1000 psi (6.9 MPa) prestress level investigated in this study is approximately 50 percent greater than the ACI value. Further studies are thus needed to determine if the ACI design recommendations are appropriate for higher values of prestress across the connection.

Based on the experimental shear strengths of 22 specimens (excluding Specimens A-1, A-12, A-13, C-1, C-2 and B-1), an average value of coefficient of friction is determined to be equal to 1.47 and has a standard deviation of 0.29.

Comparison With PCI Design Equation

The PCI Design Handbook[15] recommends the following equation for determining the shear-friction reinforcement:

$$A_{vf} = \frac{V_u}{\phi f_y \mu_e} \qquad (7)$$

where

V_u = applied factored shear force

parallel to the crack plane
ϕ = capacity reduction factor

$$\mu_e = \frac{1000\,\lambda^2 A_{cr}\;\mu}{V_u} \qquad (8)$$

The other variables have been previously defined. For purposes of this comparison, the ϕ value is taken as 1.0 and the grout is assumed to be normal weight concrete.

The PCI limits shear strength to $0.25 f_c' A_{cr}$ or $1000\,A_{cr}$ with a λ value of 1.0. Since the grout compressive strength, f_c', exceeds 4000 psi (27.6 MPa), the maximum shear strength is limited to $1000\,A_{cr}$. Assuming that the applied shear force equals the limiting value of $1000\,A_{cr}$, the value of the effective coefficient of friction, μ_e, becomes 1.0 for all cases. Consequently, with the exception of the higher upper limit on shear strength, the PCI and ACI equations yield identical results.

The product $A_{vf} f_y$ in Eq. (7) is replaced with the prestress force and the shear strength is expressed in terms of nominal shear stress. The results are presented in Table 2 and the variation of shear strength, as predicted by Eq. (7), with prestress is shown in Fig. 6.

The PCI equation underestimates shear strength at the lower prestress levels, and the difference between the experimental and calculated strength decreases with increased prestress. The shear strength predicted by the PCI equation will be more conservative if the grout is considered as lightweight concrete.

The PCI method overestimates the shear strength of the C-series specimens without shear keys. For the concrete to concrete interface condition, the recommended value of the coefficient of friction is 0.6 and the maximum value of V_u is $800\,\lambda^2 A_{cr}$.[15] Utilizing these values, the effective coefficient of friction is 0.75. Thus, the PCI approach predicts higher strengths for concrete to concrete interface condition than does the ACI approach.

Another comparison to the PCI design method is to compare the experimental values of the coefficient of friction with the values of the coefficient of friction recommended by the PCI. The effective coefficient of friction, μ_e, for each specimen is computed using the observed shear strength of the specimen, a λ value of 1 and the prestress across the connection. Experimental values of the coefficient of friction, μ, are determined using the experimental values of the effective coefficient of friction.

The average value of the coefficient of shear friction, μ, is computed as 1.25 and has a standard deviation of 0.32. The variation of the values of μ and μ_e with prestress are shown in Figs. 8 and 9, respectively. The coefficient of friction is found to be nearly constant with increasing values of prestress whereas the effective coefficient of friction is found to decrease with increasing values of prestress. The variation of the coefficient of friction with the prestress may be expressed as:

$$\mu = 1.291 + 7.36\,(10^{-5}) f \qquad (9)$$

Eq. (9) has a correlation coefficient of 0.017. A value of correlation coefficient near zero can be interpreted to

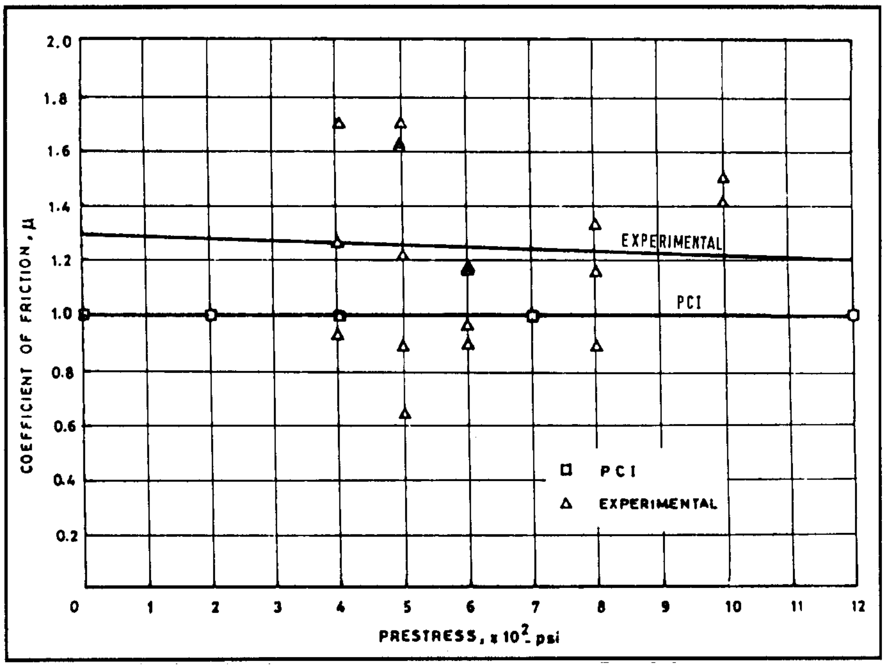

Fig. 8. Variation of coefficient of friction, μ, with prestress.

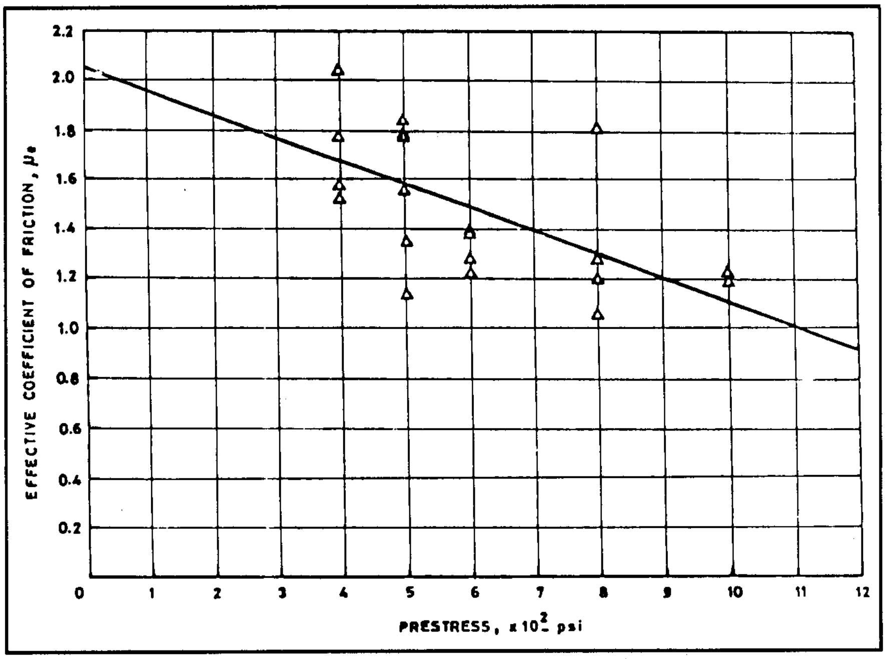

Fig. 9. Variation of the effective coefficient of friction, μ_e, with prestress.

mean that no linear association between the coefficient of friction and the prestress exists. Equations proposed by the ACI and PCI are based on the assumption that the shear strength is linearly dependent on the prestress and the coefficient of friction is constant for a particular interface condition and the type of concrete. The equation to the line of fit shown in Fig. 9 is as follows:

$$\mu_e = 2.148 - 1.124\,(10^{-3}) f \quad (10)$$

Even without any dowel action the mean experimental value of the coefficient of friction is well above 1.0. Hence, it appears reasonable to increase the coefficient of friction presented for the special case of grouted shear key connections. Moustafa has also reported that a μ value of 1 used for the evaluation of the shear strength of connection between hollow core units with extruded edges is conservative.[16]

Proposed Recommendations for Predicting the Shear Strength of Post-Tensioned Grouted Shear Key Connections

In the ACI and the PCI design methods for shear-friction, there are only four types of classifications in the interface condition: monolithic, roughened, smooth, and concrete to steel. To be classified as roughened, the interface is required to be roughened to a full amplitude of approximately ¼ in. (6.35 mm). Shear keys made to the specifications discussed provide a superior degree of roughness to the interface. Considering the extensive use of shear keys in the precast, prestressed concrete industry, they may be included as a special classification with a value of coefficient of friction between 1.0 and 1.4.

Based on the results presented herein, a value of 1.10 for the coefficient of shear-friction is considered appropriate and conservative for the shear-keyed connection using grout of strengths exceeding 4000 psi (27.6 MPa) (see Fig. 8). A value of the coefficient of friction lower than the experimental mean value of 1.3 is suggested in consideration of limited experimental data.

Prestress across the interface can be expressed as a nondimensional parameter (prestress index) by dividing prestress by the area of the cross section of the interface and the compressive strength of grout. Similarly, the shear strength can be expressed as a shear index by dividing the shear strength of the connection by the grout strength. A single line is fitted to all the data points in Fig. 10. The equation for the evaluation of shear strength proposed by Olesen is also plotted in Fig. 10 for comparison.[10] Olesen's equation is based on extensive test results of shear strength of reinforced keyed connections.

Olesen's line is found to be slightly steeper than the line fitted from the current experimental investigations. The experimental values of shear strength used by Olesen in deriving the shear strength relation include the dowel action of reinforcing bars across the connection. This is likely the reason for the increased slope of Olesen's line. However, the current experimental values compare very well with the experimental research on reinforced shear key connections used in large panel construction in the range of prestress index 0.05 to 0.15.

Mattock[13] suggested a relation for determining the shear transfer strength including the contribution of friction, the shear resistance of concrete protrusions and the dowel action of reinforcement. The Commentary to the ACI Code[14] has recommended the use of this modified shear-friction method when shear is transferred across a crack in reinforced concrete. This relation is also plotted in Fig. 10 and is found to compare well with the experimental results.

Based on the experimental investigations reported, the following equation is proposed to determine the shear strength of post-tensioned grouted shear key connections. The equation is applicable for joint thicknesses not exceeding 2 in. (51 mm) and grout strengths not less than 4000 psi (27.6 MPa):

$$v_n = 0.17\,\frac{B f_c'}{A_{cr}} + 0.65\,\frac{N_p}{A_{cr}} \quad (11)$$

where

v_n = shear strength, psi
f_c' = compressive strength of grout, psi
B = area of vertical section through grout keys, sq in.
A_{cr} = area of vertical section through joint grout, sq in.
N_p = prestressing force across the connection, lb

The first term of Eq. (11) corresponds to the contribution of grouted shear keys and the second term corresponds to the contribution of friction

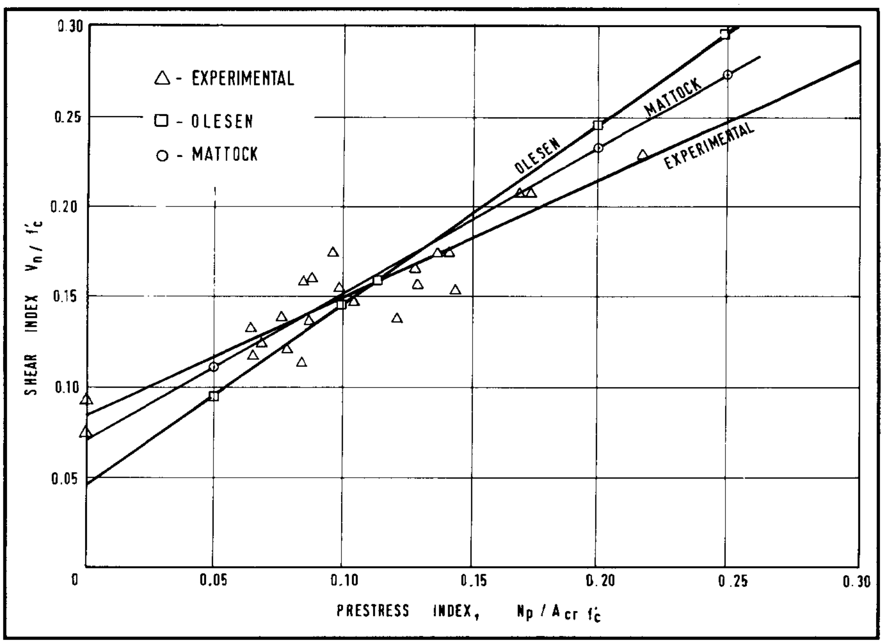

Fig. 10. Variation of shear index with prestress index.

due to the prestress, N_p. The term N_p/A_{cr} is limited to 1000 psi (6.9 MPa) in the absence of experimental data beyond 1000 psi prestress. Since the experimental data of the current investigation is comparable to that of the experimental investigation on shear strength of reinforced shear key connections, the limitations on the shape and B/A_{cr} ratios of 0.2 to 0.5 evolved by the earlier investigations may also be followed for the design of post-tensioned grouted shear key connections.

CONCLUSIONS

As a result of this experimental investigation, the following conclusions can be drawn:

1. Post-tensioning significantly improves the shear strength of grouted shear key connections which are found to exhibit a high degree of monolithic action. The proposed connection reported in this research could satisfactorily transfer shear across the connected parts.

2. The shear strength of the connection increases with increased levels of prestress. Considering the data from both the A and B test series, the shear strength increases at a rate of 0.65 times the intensity of prestress.

3. The shear strength is not significantly influenced by the distribution of prestress along the height of the connection. The shear strength of connections can be predicted based on the average compressive stress across the connection.

4. Both the ACI and PCI shear-friction methods underestimate the shear strength of post-tensioned grouted shear key connections for prestress levels less than 800 psi (5.5 MPa). The underestimate ranges from 60 percent for specimens with 400 psi (2.8 MPa) prestress and 12 percent for specimens with 800 psi (5.5 MPa) prestress.

5. The shear strength of post-tensioned grouted shear key connections can be determined using Eq. (11) with the value of the ratio of the total key area to the joint area limited between 0.2 and 0.5 and prestress levels up to 1000 psi (6.9 MPa).

6. The current ACI and PCI shear-friction design methods can be used to predict the shear strength of post-tensioned grouted shear key connections with an increased value for the coefficient of friction.

RECOMMENDATIONS

1. Considering the popular use of shear keys in precast prestressed concrete connections, they should be included as a category of interface condition in the ACI and PCI design methods. A coefficient of friction of 1.1 is considered appropriate for the shear strength evaluation of connections similar to those considered in this study.

2. The ACI maximum shear strength limitation of 800 psi (5.5 MPa) appears conservative and should be reviewed for application to post-tensioned shear key connections.

3. Studies of the effect of bending moment, dowel action and prestress losses on the shear strength of post-tensioned shear key connections are necessary.

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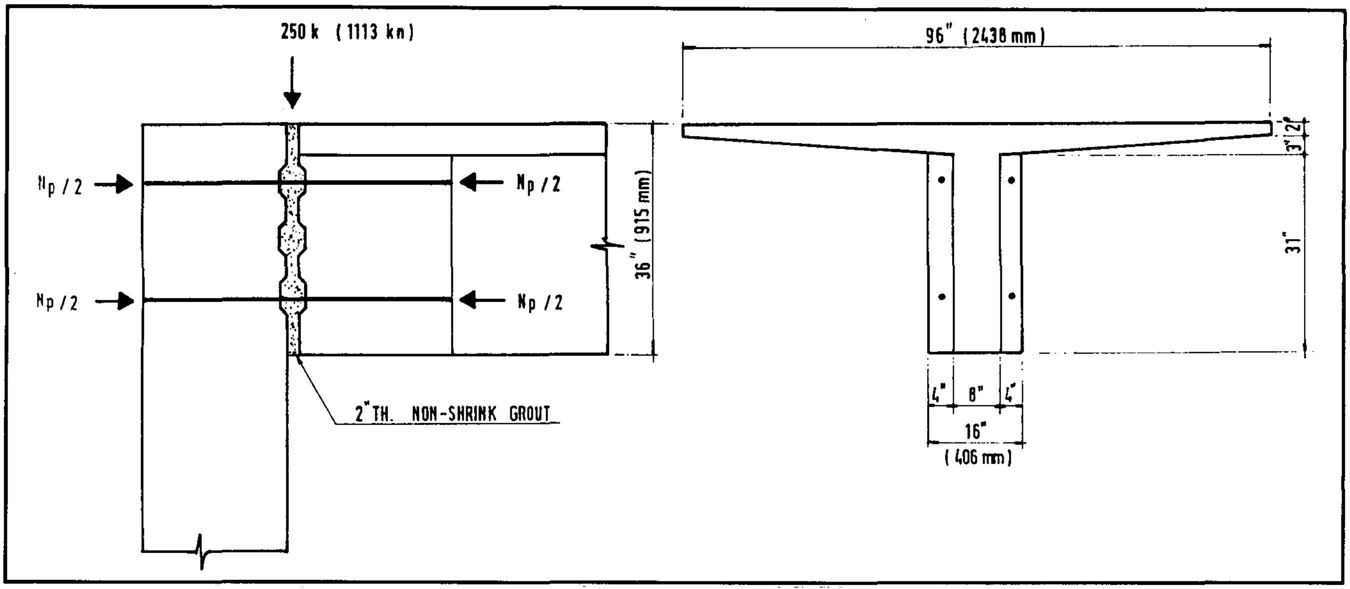

Fig. B1. Cross sections of beam-column connection used in numerical example.

APPENDIX A — NOTATION

A_{cr} = area of concrete section resisting shear along crack interface, sq in.

A_{vf} = area of reinforcement nominally perpendicular to assumed crack plane, sq in.

b_k = width of key (see Fig. 3), in.

B = area of vertical section through all concrete keys, sq in.

f = prestress per unit area of contact surface, psi

f_y = yield strength of reinforcement, psi

f_c' = compressive strength of grout, psi

N_p = prestressing force across connection, lb

t_j = thickness of joint (see Fig. 3), in.

v_c = nominal shear strength of connection, psi

v_n = shear strength of grout concrete, psi

V = total shear force, lb

V_n = nominal shear strength, lb

V_u = applied factored shear force parallel to assumed crack plane, lb

μ = coefficient of friction; coefficient of shear friction

μ_e = effective shear friction coefficient

λ = coefficient that depends on type of concrete used in connection for determining μ

ϕ = strength reduction factor

APPENDIX B — NUMERICAL EXAMPLE

Determine the prestressing force, N_p, required to transfer a design ultimate shear, V_u, of 250 kips (1113 kN) at a beam-column connection shown in Fig. B1 using:

1. ACI shear friction method
2. PCI Design Handbook method
3. Authors' proposed method [Eq. (11)]

Assume that:

Strength of nonshrink grout, $f_c' = 5$ ksi (34 MPa).

Area of shear key, B, equals one-half of the total shear transfer area.

1. ACI shear friction method

From Eq. (6):

$V_n = A_{vf} f_y \mu$

$V_u = \phi V_n$
$= \phi A_{vf} f_y \mu$
$= \phi N_p \mu \quad (A_{vf} f_y = N_p)$

$N_p = V_u/(\phi \mu)$
$= 250/(0.85 \times 1)$
$= 294.12$ kips (1309 kN)

where $\phi = 0.85$ and $\mu = 1$.

2. PCI Design Handbook method

From Eq. (8):

$\mu_e = (1000\, \lambda^2 A_{cr}\, \mu)/V_u$

where

$\lambda = 1$

$A_{cr} = 36 \times 16 = 576$ sq in.

$\mu = 1$

Now, V_u is the minimum of:

$0.25 f_c' A_{cr} = 0.25 \times 5 \times 576$
$= 720$ kips

and

$1000\, A_{cr} = 1000 \times 576/1000$
$= 576$ kips

Therefore, the controlling value of 576 kips is greater than the actual shear of 250 kips. Hence:

$\mu_e = (1000 \times 1 \times 576 \times 1)/(250 \times 1000)$
$= 2.3 < 2.9$ maximum

From Eq. (7):

$A_{vf} = V_u/(\phi f_y \mu_e)$

$N_p = V_u/(\phi \mu_e)$
$= 250/(0.85 \times 2.3)$
$= 128$ kips (570 kN)

3. Proposed Eq. (11)

$v_n = 0.17\,(B/A_{cr}) f_c' + (0.65\, N_p)/A_{cr}$

Rearranging the equation:

$V_n = 0.17\, B f_c' + 0.65\, N_p$

$N_p = (V_n - 0.17\, B f_c')/0.65$

where

$V_n = 250/0.85 = 294.12$ kips, and

$B = 0.5 \times 36 \times 16 = 288$ sq in.

Therefore,

$N_p = (294.12 - 0.17 \times 288 \times 5)/0.65$
$= 76$ kips (338 kN)

A comparison of the results of the three methods is given below.

1. ACI method: $N_p = 294$ kips
2. PCI method: $N_p = 128$ kips
3. Proposed method: $N_p = 76$ kips